POTENTIAL IMPACTS AND COSTS OF HIGH AND LOW LAKE LEVEL SCENARIOS ON LAKE MICHIGAN HARBOR STRUCTURES

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1.0 INTRODUCTION

The USACE is currently undertaking a Value Engineering (VE) Study of its harbors on Lake Michigan as a component of the Lake Michigan Potential Damages Study (LMPDS). The objective of the VE study is to assess the potential impacts of different water level scenarios (as defined by GLERL, 2000) on the maintenance/repair costs for the harbor facilities.

The USACE requested that Baird undertake a review of their water level scenario impact assessments. In addition, Baird proposed, and the USACE authorized, a preliminary assessment of conceptual design alternatives to restore/replace the aging breakwater structures that protect these harbors. The objective of such designs would be to significantly reduce the requirement for future maintenance/repair of these structures. This may, in the long-term, represent a more cost-effective approach to the management of the harbor facilities than the current approach of regular maintenance/repair. In this context, restore/replace refers to the work necessary to significantly extend the useful life of the existing structures. Consideration has also been given to alternative upgrade concepts. Such concepts might be considered by the USACE or other interests in order to provide an increased level of protection (shelter) to the harbor basin, particularly in response to the high water level scenario.

This report provides a discussion of the following topics:

1. typical range in existing structure designs;

2. typical range in Lake Michigan design conditions (waves and water levels);

3. GLERL (2000) lake level scenarios and impacts on design conditions;

4. typical maintenance/repair work currently undertaken by the USACE on its harbor structures, including an assessment of the impact of different water level scenarios;

5. alternative restoration/replacement concepts and costs; and

6. alternative upgrade concepts and costs;

7. influence of lake level scenarios on longshore transport at the harbors

The information presented in this report is intended to assist the USACE in the completion of cost-benefit analyses of the current practice of regular maintenance/repair work and the alternative of structure restoration/replacement (and possibly upgrade) for its Lake Michigan harbor structures.

It is noted that the restoration/replacement and upgrade designs presented in this report are at a conceptual level, appropriate to meet the requirements of the USACE and its Value Engineering Study. Detailed, site-specific engineering analyses, possibly including physical modeling, would be required to develop final designs and cost estimates for a specific project.

2.0 TYPICAL RANGE IN EXISTING STRUCTURES

A large number of harbors were constructed on Lake Michigan (and the other Great Lakes) by the USACE in the late 1800s and early 1900s, as shown in Figure 2.1. In general, these harbors were constructed to provide sheltered water for a range in commercial and industrial interests, thereby providing safe and efficient shipping operations (i.e. vessel loading and unloading). It is noted that some of these facilities are now used as small craft harbors (i.e. recreational marinas and/or harbors of refuge), and some no longer have active commercial or industrial interests.

In general, the harbor structures (i.e. breakwaters, jetties, wharves and revetments) were composed of timber cribs and/or timber pile walls with concrete caps, sometimes with rubble placed on one or both sides of the structure. Many of these structures are over 300 m long and extend into water depths of 6 to 10 m. Most of these structures have a crest elevation in the order of +2.1 m LWD, which permits significant overtopping during storms, particularly during periods of high lake levels. Figures 2.2 and 2.3 present typical cross-sections of existing structures at Saugatuck, MI and Manitowac, WI. Similar information for other harbors on Lake Michigan is provided in Appendix A.

A wide range of maintenance/repair work has been undertaken by the USACE on many of these structures in order to address long-term deterioration and/or to improve performance (i.e. to reduce wave reflections and/or wave overtopping), including, but not limited to:

1. the placement of additional rubble on one or both sides of the structure;

2. the addition of parapet walls on the structure crests; and

3. the complete reconstruction with a new rubblemound structure or parallel steel sheet pile walls and a concrete cap over the original structure.

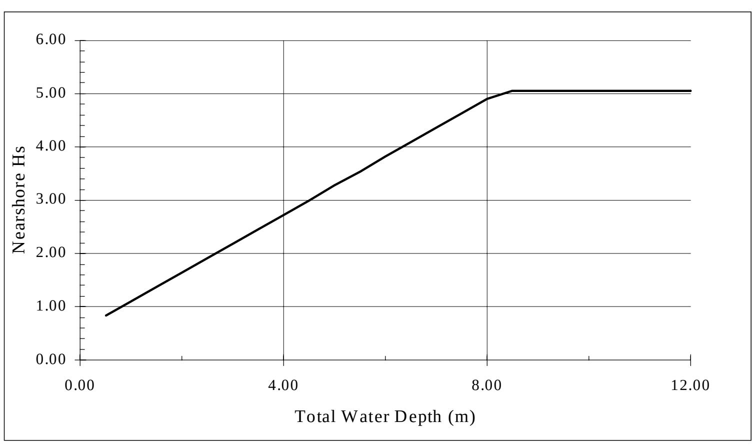
A discussion of the USACE's maintenance/repair program, and an assessment of the potential impacts of different water level scenarios on maintenance/repair requirements, is provided in Section 4 of this report.

3.0 TYPICAL LAKE MICHIGAN DESIGN CONDITIONS

3.1 Previous Wave Climate and Water Level Analyses

The primary environmental design conditions that must be considered in the design of the structures under consideration are waves and water levels. The frequency of occurrence and severity of the design waves and water levels varies around the perimeter of Lake Michigan. For example, the WIS hindcast study for Lake Michigan (USACE WES, 1991) documents the wave climate and extreme wave conditions around the perimeter of the Lake for the period 1956-87 (now extended through 1997, but not yet publicly released), while the USACE (1988) Open Coast Flood Levels report documents extreme water levels around the perimeter of the Lake. In addition, Baird has project experience, including site-specific wave climate and water level analyses, at numerous locations around the Lake. This includes detailed wave climate and water level analyses undertaken for the five prototype counties under consideration in the LMPDS (Baird, in progress). It is noted that Baird's work suggests that the WIS hindcast study may underestimate the wave climate along the south and west shores of Lake Michigan for the period 1956-87.

Based on the available information, it can be concluded that extreme offshore design wave heights around the exposed shorelines of Lake Michigan are generally in the order of Hso = 4.5 to 6 m, Tp = 9 to 11 s, assuming a return period in the order of 20 years. These wave conditions would become depth-limited in water depths in the order of 6 to 10 m. For example, Figure 3.1 presents the transformation in significant wave height due



to

Figure 3.1

Typical Nearshore Wave Transformations

shoaling and breaking (based on the methodology of Goda, 1985) assuming a 1:100 nearshore slope and Hso = 5.5 m, Tp = 10 s. In this example, breaking initiates in a total water depth of approximately 8.5 m. It is noted that refraction and diffraction effects are not included in this example.

Considering water levels, a design high water level in the order of +1.8 m LWD (including lake levels plus storm surges) is generally considered in the planning and design of new harbors and harbor improvements (for example, refer to Baird, 1994). This water level coincides with a return period in the order of 10 to 50 years depending on location, analyses methodology etc. A design low water level in the order of 0 to –0.6 m LWD would generally be considered, depending on the design issue under consideration.

GLERL (2000) has developed a number of lake level scenarios in an attempt to assess the potential range in lake levels on Lake Michigan over the next 50 years. A discussion of the impact of two of these scenarios on the typical design wave and water level conditions discussed above is presented in the next section of this report.

3.2 Impact of GLERL Water Level Scenarios

GLERL (2000) has developed five lake level scenarios through the application of hydrological modeling of the Great Lakes drainage basin in an attempt to assess the potential range in lake levels on Lake Michigan over the next 50 years. The hydrological modeling has been undertaken over a 100 year period (1900-99) using various assumptions concerning precipitation and evaporation process. The first and last 25 years are considered as “warm-up” and “warm-down” periods, with the middle 50 years representing the actual simulation period. The different scenarios are listed below:

1. Similar Wet and Dry
2. More Wet and Dry
3. More Wet Years (“High”)
4. More Dry Years (“Low”)
5. More Extreme Wet and Dry

Figure 3.2 presents the five different lake level scenarios as well as the historical record on Lake Michigan for the 50 year simulation period. The third (“High”) and fourth (“Low”) lake level scenarios have been considered by the USACE in its assessment of the impact of lake level on the scope and magnitude of repair/maintenance of harbor structures on Lake Michigan.

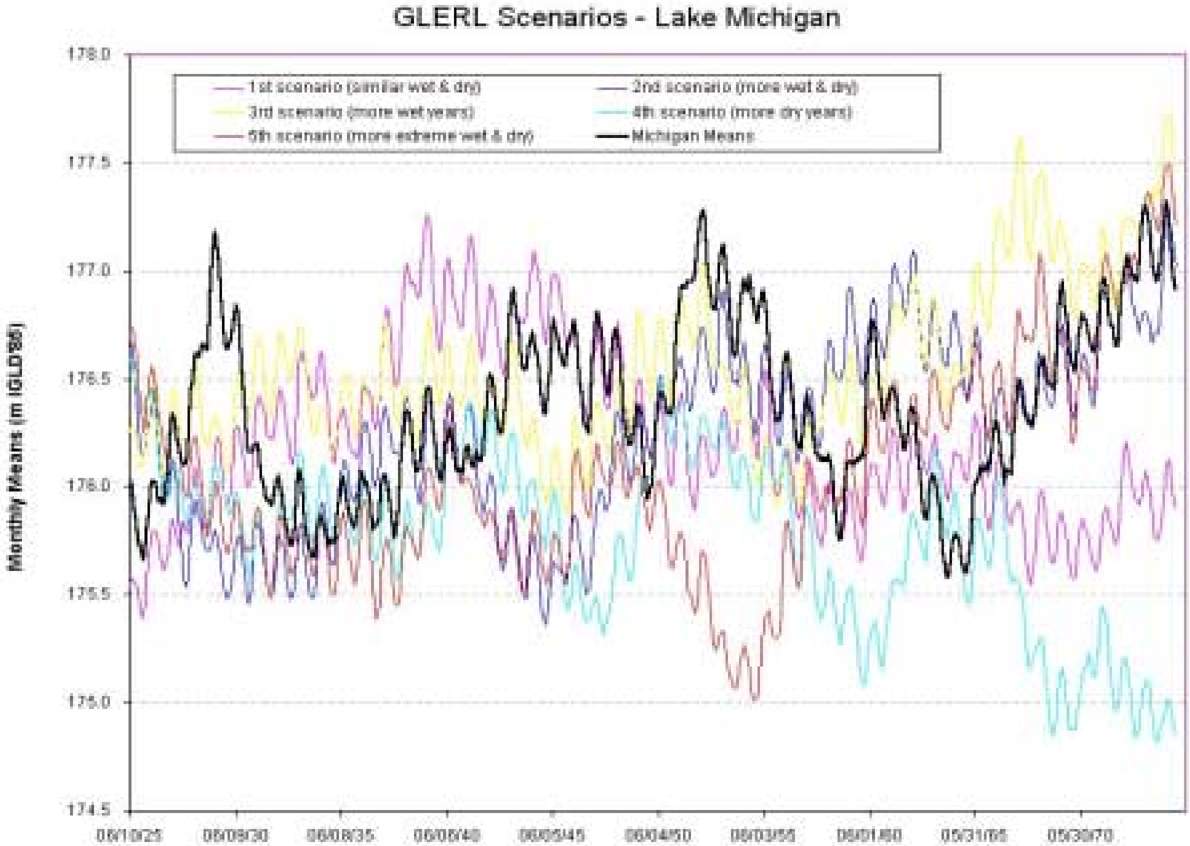


Figure 3.2 – The Five Lake Level Scenarios from GLERL

Table 3.1 summarizes the changes in extreme high and low lake levels for the "high" and "low" lake level scenarios relative to the "base" scenario (1925-74 historical record) on Lake Michigan.

Table 3.1
Impact of GLERL Lake Level Scenarios on Extreme Lake Levels

Lake Level Scenario	Change in High Lake Level (m)	Change in Low Lake Level (m)
High	+0.68	+0.55
Low	-0.44	-0.51

Based on this information, it is concluded that GLERL's "high" lake level Scenario 3 would result in a 0.5 to 0.7 m increase in the high and low lake levels that would be considered in the design of coastal structures on Lake Michigan. Similarly, GLERL's "low" lake level Scenario 4 would result in a 0.4 to 0.5 m decrease in the high and low lake levels that would be considered in the design of coastal structures on Lake Michigan. Assuming that the magnitude and frequency of occurrence of storm surges does not change, these values would also apply to the design water levels (i.e. including lake level fluctuations and storm surges) to be considered in the design of coastal structures.

In the context of the design of coastal structures, higher water levels would generally result in more severe nearshore wave conditions (due to the increased water depth and decreased breaking) and more wave overtopping, whereas lower water levels would generally result in less severe nearshore wave conditions and less wave overtopping. Based on the example presented earlier in Figure 3.1, a 0.5 m change in water depth would result in a 0.3 m change in the significant wave height within the breaking zone. A change in the design wave height of this magnitude would not have a significant effect on the structural design of a typical breakwater structure on Lake Michigan. However, the combined effect of a higher design water level and an increased design wave height might be an important consideration in defining the crest elevation of the breakwater structure to provide an acceptable level of protection to a particular harbor facility.

4.0 MAINTENANCE/REPAIR PRACTICES

As noted earlier, a wide range of maintenance/repair work has been undertaken by the USACE on many Lake Michigan harbor structures in order to address long-term deterioration and/or to improve performance (i.e. to reduce wave reflections and/or wave overtopping). Typical maintenance/repair work has included:

1. the placement of additional rubble on one or both sides of the structure;

2. the addition of parapet walls on the structure crests; and

3. the complete reconstruction with a new rubblemound structure or parallel steel sheet pile walls and a concrete cap over the original structure.

It is understood that the historical experience of the USACE in maintaining/repairing these structures suggests the following general trends in damages with respect to lake level:

1. High Lake Levels

1.1 rubble on the rear (harbor) side of these structures is more prone to damage (erosion) due to increased wave overtopping

2.1 increased wave agitation in harbor basins due to increased wave overtopping

2. Low Lake Levels

2.1 rubble on the front (lake) side of these structures is more prone to damage (erosion) due to increased wave reflections

2.2 increased deterioration of the timber cribs due to increased exposure to air above the water

In general, these damages are addressed through localized maintenance and repair work, such as the addition of rubble and the removal/replacement of damaged sections of cribwork. This work is generally undertaken by the USACE based on an assessment of need and available funds for each harbor.

The USACE has developed a template to summarize the potential impacts/costs of different lake level scenarios on Federal interests at the harbors, including both dredging and coastal structures (breakwaters, jetties, revetments, confined disposal facilities, etc.). USACE area office staff have undertaken an assessment of the potential impacts/costs of different lake level scenarios (GLERL low and high lake level scenarios, relative to the base case) at seven Lake Michigan harbors, including four in Wisconsin (Manitowac, Port Washington, Sheboygan and Two Rivers) and three in Michigan (Grand Haven, Holland and Saugatcuk). Baird has reviewed the USACE assessments for these harbors, and has a number of general comments, as presented below. ***Specific questions and comments regarding the individual harbor assessments are provided in Appendix B.***

General

1. quantification of the potential impacts/costs of different lake level scenarios on harbor damages and maintenance/repair requirements is difficult, and relatively subjective

2. different experts may develop different opinions and reach different conclusions from those reached by the USACE staff

Dredging

3. the requirement for dredging is principally defined by the draft of the largest vessels using the harbor, as well as the minimum expected water level and the maximum expected wave action); a change in the characteristics of the vessels using the harbor may affect the requirement for dredging; it is noted that several of the harbors are no longer used by commercial shipping interests, and several have never been dredged to their authorized project depths; as such, their may be no requirement to maintain the authorized project depth

4. the requirement for maintenance dredging for the Wisconsin harbors is generally reduced as the water level increases from the low to the base to the high lake level scenarios; however, it is not clear how the decrease in the frequency of maintenance dredging has been defined; for the Michigan Harbors the opposite is true: dredging increases at higher lake levels – this is supported by the findings of Section 6.

5. a 2 ft (0.6 m) increase in the project depth is proposed under the low lake level scenario; as noted in Table 3.1, the “design” low lake level decreases by 0.5 m (1.5 ft) under this scenario, which suggests that a 1.5 ft increase in the project depth may be adequate

Breakwaters

6. an increased volume and/or frequency of placement of riprap on the rear (sheltered) side of the breakwaters is proposed for the Wisconsin harbors as the water level increases from the low to the base to the high lake level scenarios; this appears reasonable, considering the increase in wave overtopping expected under higher water levels; however, accurate quantification of the damage, and the associated maintenance/repair requirements, would require a physical model investigation; it is noted that no water level impact is suggested for the Michigan harbors; this is an example of different professional opinions on a particular issue

7. different opinions are also apparent regarding the impact of lake levels on the requirement for additional riprap on the front (exposed) side of the breakwaters; in some cases, an increased volume and/or frequency of placement is proposed under higher water levels, while in other cases, the opposite is proposed, while in other cases, there is no difference; Baird notes that wave reflections from the breakwaters may increase somewhat at lower water levels, but that the incident wave heights will be reduced, at least in the breaking zone; intuitively, the impact of these two changes on the requirement to add riprap to the exposed sides of the breakwaters would be opposite; accurate quantification of the damage, and the associated maintenance/repair requirements, would require a physical model investigation

8. a 4 ft (1.2 m) high parapet wall is generally proposed under the high lake level scenario in order to address increased wave agitation due to wave overtopping; Baird's experience suggests that a parapet wall may not be particularly effective at reducing wave overtopping and associated wave agitation levels under extreme storm conditions; further, it is noted that wave agitation levels may not be an important issue for commercial harbors and shipping operations, but will be an important issue if the basin is being used as a small craft harbor or harbor of refuge; as such, breakwater improvements to reduce wave overtopping may only be warranted for small craft harbors

9. it is noted that full replacement of the breakwater cribs is proposed under all water level scenarios for the Wisconsin harbors, but under no scenarios for the Michigan harbors; for the Wisconsin harbors, this work is proposed earlier under lower water level scenarios, presumably due to the increased height of the cribs exposed above the water level; however, it is not clear how the change in timing for replacement work was defined

10. for the Wisconsin harbors, the requirement for breakwater cap repairs varies from none to grout to full crib replacement, perhaps due to several factors, including the original structure design, the existing structure conditions and the water level scenario (specifically the relationship between the lake level and the crib/cap elevation); it is noted that the lake levels fluctuate considerably within each scenario, and that similar breakwater cap repairs might be required under the different water level scenarios; for the Michigan harbors, only minor (localized) repairs are proposed, under the low and/or high lake level scenarios (the base scenario was not considered)

11. flanking is generally only an issue under the high water level scenario; this appears to be a reasonable assumption

In summary, there is a considerable difference in the lake level impact assessments developed by the USACE area office staff for the Wisconsin and Michigan harbor structures. These differences may be partially the result of different types of structures and different conditions of structures. Specifically, many of the old timber structures in Michigan have been recently replaced with steel sheet pile structures (eg. holland jetties). In addition, and as noted above, it is difficult to quantify the potential impacts of lake levels on the damage and associated maintenance/repair requirements for harbor structures. As such, the differences may also be partially the result of different professional opinions.

The local experience/expertise of the USACE area office staff is invaluable in the lake level impact assessments, but is subject to two important limitations:

12. the assessment may differ depending on the individual and his/her experience, and

13. the experience/expertise is lost with the departure of the individual.

It is our opinion that the only way to improve the quantification of the potential impacts of lake levels on damages to harbor structures would be through detailed physical model investigations. However, this level of assessment is probably not warranted in the context of the LMPDS.

Section 5 presents alternative design concepts that might be considered to provide pro-active “engineered” solutions to the ongoing damage/deterioration of the harbor structures, and/or to improve the performance of the existing structures. These types of solutions represent an alternative to the ongoing “ad-hoc”, reactive maintenance/repair work currently undertaken by the USACE.

5.0 RESTORATION/REPLACEMENT AND UPGRADE CONCEPTS

5.1 Restoration/Repair Concepts

Two restoration/repair concepts have been considered at this time, as noted below:

R-1 Rubblemound Protection over Existing Structure

R-2 SSP Walls and Concrete Cap over Existing Structure

Figures 5.1 and 5.2 present schematic diagrams of these two concepts, which have been developed on the basis of Baird's extensive experience in the planning, design and construction of coastal structures. These concepts are intended to provide long-term structural stability under the assumed design conditions (20 year waves and water levels), with the objective of significantly reducing future maintenance/repair costs.

The first concept involves the construction of a conventional rubblemound revetment structure on one or both sides of the existing structure, and requires the use of relatively large armour stone to resist the design wave conditions. This concept has been implemented by the USACE at Holland, MI. The second concept involves the construction of new, parallel steel sheet pile walls on either side of the existing structure, with granular fill and a new concrete cap. This approach has been implemented by the USACE at several harbors on Lake Michigan. Both concepts might require some demolition of the existing crib structure to prevent future deterioration/settlement within the new structure.

The primary objective of these restoration/replacement concepts is to extend the functional life of the existing structure, and to minimize the requirement for future maintenance/repair. Neither concept is intended to provide a significant increase in the performance of the structure (i.e. the level of protection it provides to the harbor). It is noted that the original structures, which were designed and constructed in the late 1800s and early 1900s, were based on design lake levels that are much lower than those now considered in the design of coastal structures on Lake Michigan. The existing structures might not meet existing standards with respect to wave agitation levels under currently adopted design wave and water level conditions. However, they may provide an acceptable level of performance depending on the nature of operations within the harbor. As noted earlier, an improved level of performance may only be required/warranted for small craft harbors. Alternative upgrade concepts are discussed in Section 5.2.

The impact of different water level scenarios on the design and cost of these breakwater restoration/replacement concepts is considered to be small, if not insignificant, assuming that it is not necessary to improve the level of performance provided by the existing structures. As noted earlier, the change in nearshore design wave height (Hs) associated with the high and low lake level scenarios would be in the order of +/-0.3 m. This would not have a significant impact on the design of the armour layer or steel sheet pile walls. If the existing level of performance (with respect to shelter provided to the harbor) must be maintained under the high lake level scenario, than an increase in the breakwater crest elevation in the order of 1 m would likely be required.

Both design concepts have advantages and disadvantages that may impact the selection of the most appropriate concept for a particular project. For example, the rubblemound design concept will result in lower wave reflections. On the other hand, the steel sheet pile wall/concrete cap concept may be more suitable where public access is desired/required. The selection of the most appropriate restoration/replacement concept for a particular project would require more detailed site specific analyses, including consideration of construction and cost factors. A preliminary opinion of probable construction costs is provided in Section 5.3. Detailed engineering analyses, possibly including physical model investigations, would be required to develop final designs for implementation for a particular project.

5.2 Upgrade Concepts

Four potential upgrade concepts have been considered at this time, as noted below:

U-1 Rubblemound Protection over Existing Structure

U-2 SSP Walls and Concrete Cap over Existing Structure

U-3 Rubblemound Berm Adjacent to (Behind) Existing Structure

U-4 Rubblemound Breakwater Inside (Behind) Existing Structure

Figures 5.3 to 5.6 present schematic diagrams of these concepts, which have been developed on the basis of Baird's extensive experience in the planning, design and construction of coastal structures. These concepts are intended to provide a significant increase in the level of protection provided to the harbor basin (consistent with the requirements of a small craft harbor) and long-term structural stability under the assumed design conditions (20 year waves and water levels) with limited maintenance/repair costs.

The first two upgrade concepts are similar to the repair concepts discussed in Section 5.1, with the geometry of the structure (in particular, the crest elevation and width) increased to provide greater protection to the harbor. These upgrade concepts are intended to improve the performance of the existing structure, while also extending its functional life.

The third and fourth upgrade concepts involve the construction of new rubblemound structures inside the existing structure in order to increase the level of protection provided to the harbor. The shelter provided by the existing structure reduces the design requirements (cross-section geometry and armor stone size) for the new structures. However, these concepts are dependent upon the shelter provided by the existing breakwater structure. As such, the existing structure must be in good condition and be expected to remain in good condition for the design project life.

A fifth upgrade concept might consist of a new rubblemound breakwater constructed lakeward of the existing breakwater. This concept would provide an increased level of protection to the harbor, and might also extend the useful life of the existing breakwater structure to some degree. However, previous investigations by Baird (1994, 1998) suggest that this is not a cost-effective approach to increasing the level of protection provided to typical Lake Michigan harbor basins. As such, it has not been considered any further.

Each of these design concepts has advantages and disadvantages that may impact the selection of the most appropriate concept for a particular project. The selection of the most appropriate upgrade concept for a particular project would require more detailed site specific analyses, including consideration of construction and cost factors. A preliminary opinion of probable construction costs for the different concepts is provided in Section 5.3. Detailed engineering analyses, possibly including physical model investigations, would be required to develop final designs for implementation for a particular project.

It is noted that Baird (1985, 1990, 1994, 1996, 1998) has considered these design alternatives for several harbor upgrade projects on Lake Michigan, including projects at Racine, Sheboygan, Milwaukee, Gary and Hammond. A range in final design solutions has been adopted for these projects, illustrating the site and project specific nature of this type of work.

5.3 Preliminary Opinion of Probable Construction Costs

Preliminary cost estimates have been developed for each of the restoration/replacement and upgrade concepts presented above. These cost estimates are based on the structure geometries presented in Figures 5.1 to 5.6 and unit costs derived from recent project experience at several locations around Lake Michigan. The assumed unit costs are provided in Table 5.1, while the cost estimates are summarized in Table 5.2. These cost estimates are intended for comparison purposes only, and should not be used to establish budgets for a particular project.

Table 5.1
Assumed Unit Costs

Description	Unit Cost
Bedding/Core/Fill Stone	\$25/T
Filter Stone	\$35/T
Wide-Graded Armor Stone	\$35/T
Narrow Graded Armor Stone	\$40/T
Steel Sheet Piling	\$30/ft^2
Cast-in-Place Reinforced Concrete	400/yd^3

Table 5.2
Preliminary Opinion of Probable Construction Costs
(quantities based on assumed water depth of -12 ft LWD)

Design Concept	Estimated Cost per Lineal Foot (\$/ft)
R-1 Rubblemound Revetment Adjacent to Existing Structure	***\$2,300/ft***
R-2 SSP Walls and Concrete Cap over Existing Structure	***\$3,100/ft***
U-1 Rubblemound Breakwater over Existing Structure	***\$2,700/ft***
U-2 SSP Walls and Concrete Cap over Existing Structure	***\$4,500/ft***
U-3 Rubblemound Berm Behind Existing Structure	***\$2,350/ft***
U-4 Rubblemound Breakwater Inside Existing Structure	***\$1,900/ft***

These estimates suggest that the rubblemound design concepts are less expensive than the steel sheet pile design concepts. In addition, it is noted that the rubblemound upgrade concepts (U-1, 3 and 4) are similar in cost to the rubblemound restoration/replacement concept (R-1). However, it is important to emphasize that these are preliminary cost estimates, based on a number of assumptions regarding design conditions, required performance and unit costs. These factors may vary significantly from one project to another. In addition, there are numerous considerations, other than cost, that may affect the selection of a preferred design alternative for a particular project.

The USACE has implemented Concept R-1 at Holland, MI, and Concept R-2 at several harbors on Lake Michigan. Baird has been involved in the planning, design and construction of several harbor upgrade projects on Lake Michigan. For example, Concept U-3 was used along the north breakwaters at Racine and Sheboygan harbors in order to reduce wave overtopping/transmission, while Concept U-4 was used along the north breakwater at McKinley Harbor in order to reduce wave agitation in the McKinley harbor basin.

As noted earlier, more detailed engineering analyses would be required to select the most appropriate design concept, and to develop final designs and cost estimates for the selected concept, for a particular project.

6.0 INFLUENCE OF LAKE LEVEL SCENARIOS ON LONGSHORE TRANSPORT

This section presents an investigation of the influence of lake level on longshore sand transport rates and the resulting impact on channel sedimentation and dredging. The investigation is focused on the harbors at Saugatuck, MI and Manitowoc, WI.

There are two sources of harbor channel sedimentation:

1. Sand transported in the littoral zone along the shore and deposited in the outer part of the entrance channel near the lakeward end of the jetties and beyond; and,

2. Silt and sand carried by rivers that often flow out between the harbor jetties. This sediment is often deposited inland or upstream of the jettied entrance channels where river flow velocities drop off. This could occur in the bay formed by arrowhead breakwaters (e.g. Maritowac) or in the lakes formed by the drowned river mouths that are prevalent on the Michigan side of Lake Michigan (e.g. Kalamazoo Lake at Saugatuck).

While the lake level may influence the location of deposition of the river-derived sediment (through changes to backwater effect) the quantity of deposition is not influenced by the lake level.

The sand carried along the shore will be the focus of this investigation. The cross-shore distribution of longshore sand transport rates has been estimated using the COSMOS numerical model operating with the GIS frameworks of the Flood and Erosion Prediction System developed for the LMPDS. The COSMOS model predicts the cross-shore distribution of longshore currents, which when combined with sediment stirred by orbital velocities and breaking waves leads to longshore transport.

In the investigation of the coastal processes for the study sites, it was shown using data from Little Sable Point that the barred profiles that are a ubiquitous feature on the generally sand-rich east coast of Lake Michigan shift with changes in lake level (Baird, 1998). The bars shift upward and landward during rising lake levels. The extent of the upward shift was shown to be similar to the rise in lake level for a given time period as expected with the Bruun Rule response of profiles to changing water levels. The shoreward shift is required to balance offshore deposition (caused by the upward shift) with erosion at the shore.

Information on the actual profiles at Saugatuck and Manitowoc under low, normal and high water conditions was unavailable. The SHOALS survey completed in November 1999 featured a low lake level condition (lake level of approximately 176.0 m IGLD). Therefore it was assumed that this profile was representative of low lake level conditions. This makes the assumption that the profile had fully responded to this low lake level. The lake level dropped to approximately 176 m (IGLD) through 1999 and several storms were experienced during this period making this assumption reasonable. Unfortunately, the SHOALS data did not include coverage in the immediate vicinity of the harbor mouth and therefore the actual conditions are unknown.

Profiles for a moderate lake level (176.5 m IGLD) and a high lake level (177 m IGLD) were determined by shifting the 1999 profiles upward by 0.5 m and 1 m respectively. These two profiles were shifted shoreward by 30 m and 60 m respectively relative to the 1999 profile to approximately balance erosion and deposition. The result is shown in Figure 6.1 for Saugatuck.

The next step was to complete longshore transport simulations for the High (Scenario 4), Base or moderate (Scenario 1) and Low (Scenario 3) wave and lake level conditions with the respective profiles. This is a gross simplification as the profile (and particularly the bar positions across the profile) is always shifting in response to changing levels (whereas here the position has been assumed to be static for each simulation). However, this exercise serves to demonstrate the longshore transport patterns during different lake level conditions.

The results of these simulations are shown individually in Figure 6.2, 6.3 and 6.4 and as a group in Figure 6.5. Each figure includes two components of longshore transport - northward and southward. The net transport at Saugatuck is about 61,000 m^3/y towards the south based on the balance between 227,000 m^3/y transported to the south and 166,000 m^3/y transported to the north. The distribution features three peaks one near the shoreline (in the swash zone) and one each over the inner and outer bars. Each of Figures 6.2 to 6.5 also show the nearshore profile, the approximate extent of the pier and the channel dredge depth (16 ft or 4.8 m in the case of Saugatuck). Figure 6.5 shows how the position of these three peaks change with the different lake level scenarios (and associated profiles). It is likely that the two inner peaks are intercepted at least in part, by the harbor jetties, possible trapping the sand either temporarily or permanently in the fillet beaches adjacent to each pier. Under low water the peak over the inner bar moves close to the end of the pier.

The peak over the outer bar varies considerably in magnitude for the three lake level scenarios. This is somewhat artificial as it results from more waves breaking over the high lake level outer bar during both low and high lake levels that are experienced during the high lake level scenario. If we focus on the average lake level scenario, the northward component of transport over the outer bar is about 44,500 m^3/y and the southward component is 72,000 m^3/y. It is likely that most of this transport bypasses the entrance channel and does not lead to deposition (in other words it is likely that this bar bisects the entrance channel most of the time).

Making the assumption that a bar does bisect the entrance channel beyond the jetties (without direct hydrographic evidence), the volume of a bar across the flared entrance channel at Saugatuck ranges from 6,500 to 39,000 m^3 depending on the position (which is influenced by lake levels). The dredging records for Saugatuck are shown in Figure 6.6. Between 8,000 and 29,000 m^3 was removed in 1990, 1993 and 1997. This corresponds well to the quantity required to remove the outer bar within the entrance channel. It is likely that this bar re-establishes very quickly (probably within a year) based on the relatively high transport rates over this outer bar.

Sand which enters the entrance channel from longshore transport (particularly from the inner bar) may also be flushed by the flow through the jetties. Flows on the Kalamazoo River emptying through Saugatuck Harbor into Lake Michigan range from 14 to 140 m^3/s. These flows result in channel cross-section averaged velocities between 0.1 and 0.9 m/s. During high flows these velocities would be capable of flushing sediment from the entrance channel. Therefore, the need to dredge may also be related to the frequency of occurrence of large flushing flood events. For example, an extended period of low flows in the river and the associated low flushing may result in the build of sediment at the mouth from littoral processes. At Grand Haven the Grand River produces cross-section averaged flows of 0.02 to 0.22 m/s through the wider jettied channel at this location. The flow generated by the main river emptying into the harbor at Holland generates cross-section averaged flows of 0.13 to 0.58 m/s through the pierhead of the arrowhead jetties. The influence of lake level on the range of flow velocities at each channel was assessed and found to be relatively small. These estimated flushing capacities compare well to the relative quantities of dredging in the outer harbor areas with the most dredging at Grand Haven where flushing velocities are lowest and the least at Saugatuck where the velocities are highest.

Another factor that must be considered is the position of the outer bar due to cross-shore sand transport. Some harbor sedimentation problems are related to the movement of bars into the channel caused by cross-shore transport. As shown in Figures 6.1 and 6.5 the bar moves close to the entrance channel proper within the jetties during high lake levels. It is possible that under same conditions this bar may move directly into the jettied channel. Also, as noted above the flushing capacity is reduced during high lake level conditions.

Therefore, based on this qualitative assessment it would appear that high lake conditions might lead to the most critical situation at Saugatuck. A more detailed analysis including the application of a 2-dimensional numerical model of wave and river generated flows would have to be applied. This investigation also leads to the conclusion that the influence of lake levels on dredging requirements will be very harbor specific, depending on the length of the piers, the channel depth the river discharge characteristics and the relative position and shape of the nearshore profile.

Figures 6.7 and 6.8 compare the dredging quantities per year to lake levels for Holland and Grand Haven Harbors, respectively. A pattern of slightly greater dredging during high lake levels is observed, particularly for Grand Haven.

From the perspective of the LMPDS investigation, these findings of a tendency for dredging to increase with higher lake levels agree with the information provided by the Grand Haven area office of the USACE for the harbors at Holland, Saugatuck and Grand Haven.

From a VE perspective, the findings suggest that the frequency of dredging is possibly more related to a programmed dredging frequency than an actual requirement. For example, at Saugatuck it is possible that the outer nearshore bar is quickly re-established after dredging. A more detailed assessment of these issues including an analysis of hydrographic surveys and 2D modeling of waves and currents may lead to recommendations to optimize the dredging operations.

The frequency of dredging is also closely related to the nourishment of the shoreline adjacent to the harbors whether part of a Section 111 program or not. Since approximately 1990, all of the sand dredged from the "outer" section of the navigation channels has been placed directly on the adjacent beaches. The placement of dredged sand on adjacent beaches is part of the harbor maintenance dredging program and as such is not at an added cost to the Section 111 Program. However, at some locations the nourishment from dredging is occasionally complemented by sand and gravel that is trucked in and placed on the beaches adjacent to the harbors. A review of the Section 111 Program summary (provided by USACE Detroit) indicates that trucked sand is placed during periods of high lake levels. There are no records of sand trucked to the beaches adjacent to the Saugatuck Harbor and Saugatuck is not part of the Section 111 Program. Holland received trucked sand under the Section 111 Program in 1978 (122,000 m^3) and 1986 (77,000 m^3), in the latter case it was referred to as "Emergency Section 111". These two years are significant as they are either just following (1978) or during (1986) periods when the lake levels exceeded 177 m (IGLD). Grand Haven received 127,000 m^3 of trucked sand as part of an Emergency Section 111 effort in 1986. Therefore, with respect to the costs associated with the Section 111 program, it may be assumed that Section 111 harbors such as Grand Haven and Holland will receive approximately 125,000 m^3 of trucked sand when the level of Lake Michigan exceeds 177 m (IGLD). A review of the GLERL lake level scenarios (Figure 3.2) shows that for the High Scenario 3 trucked nourishment will be required two times corresponding to Years 1966 and 1974 in Figure 3.2 which translates to Years 1 and 9 in a 50 year scenario run in the reverse direction. The Base Scenario 1 would require trucked nourishment in Year 1940 or Year 35 within a 50 year scenario run in the reverse direction. The Low Scenario 4 would not require any trucked nourishment.

APPENDIX A

DRAWINGS OF LAKE MICHIGAN HARBOR STRUCTURES